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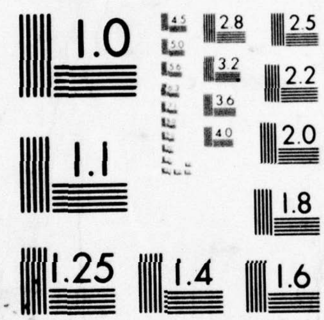

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STRUCTURAL RESEARCH SERIES NO. [illegible]

DESIGN OF THICK-WALLED MULTIPLE OPENING CONDUITS TO RESIST LARGE DISTRIBUTED LOADS

By
W. L. GAMBLE

A Report on the Study
"Investigation of Multiple Opening
Concrete Conduits"
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The design of multiple opening reinforced concrete conduits suitable for use as waterways through and under large earth-fill dams is discussed. The loadings considered are given values, and are not a subject of this investigation. Vertical overburden pressures up to 30 k/ft^2 (square ft), corresponding to about 250 ft. of fill, are considered.

The resultant structures have members with low span/depth ratios, and the analysis and design of these structures is discussed, in a number of over

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steps. The idealization of the thick-walled structure into a frame of linear elements is discussed, with particular attention to the joint areas.

The analysis of the idealized frame is then considered. After the basic analysis is done, moments, shears, and thrusts must be found for the critical cross sections, and the selection of these sections is discussed.

A criterion for the shear strengths of the deep members used in the conduits is presented. This is based on tests of models done earlier in the investigation, and has been checked against the results of other deep beam tests.

The flexural reinforcement and the detailing of the flexural steel are discussed.

Table of Contents

1. Introduction

1.1 Objective and Scope

This report is a guide for the designer faced with the problem of the analysis, proportioning, and detailing of multiple opening rectangular reinforced concrete conduits subjected to very heavy loadings. It was developed during an investigation of conduits suitable for use under high earth-fill dams, and may be useful for other purposes, including box culverts under highway or railway fills. The cross section of a possible structure is shown in Fig. 1.

The design process can be divided into several phases, as follows:

1. Determine loadings,
2. Idealize structure into model for analysis,
3. Analyze to find controlling combinations of moment, thrust, and shear for each member,
4. Proportion sections for forces, and
5. Detail structure, including connections.

This may be an iterative process, especially involving steps 2, 3, and 4.

The background for the following recommendations can be found in three reports (1, 2, 3)* which describe the results of a series of tests of eight three-opening conduit specimens and an analytical study of the same structures. The analysis used the finite element technique, taking into account progressive cracking and nonlinear stress-strain properties of the materials.

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* Numbers in parentheses refer to entries in List of References.

This study was undertaken to provide information for the design of structures with 250 to 300 ft of fill over them. The current design procedures, such as outlined in Ref. 4, were originally developed for fill depths up to 75 ft, and these procedures appear to lead to excessive member thicknesses for larger loads. Reference 4 is partially based on work done at the University of Illinois at Urbana-Champaign before 1960 (5, 6), and this study can be viewed as an extension of that work although there is no formal continuity.

The remainder of this report will be divided into chapters dealing with each of the five design steps listed above.

1.2 Administrative Organization and Acknowledgements

The administration of the contract for the Corps of Engineers has been under the supervision of Wendell E. Johnson, Joseph H. Caldwell, and Homer B. Willis, who have served successively as Chief, Engineering Division, Civil Works, Office of the Chief of Engineers. Direct contact between the Engineering Division and the University has been through Charles F. Corns, Howard W. Goodhue, Keith O. O'Donnell, and Lucian G. Guthrie, and appreciation is expressed for their continued interest and many helpful suggestions.

At the University of Illinois at Urbana-Champaign, the work has been under the administrative direction of D. C. Drucker, Dean of the College of Engineering, R. J. Martin, Director of the Engineering Experiment Station, and N. M. Newmark and C. P. Siess, successive heads of the Department of Civil Engineering. Professor W. L. Gamble has provided overall supervision of the project and of the experimental phase of the investigation.

Professor Bijan Mohraz, now at Southern Methodist University, directed the analytical phase of the work.

Acknowledgements are due to Dr. A. E. Aktan, T. A. Broz, J. L. Hoebel, M. O. Ryan, G. C. Rucicka, Dr. M. H. Salem, and Dr. L. G. Pleimann, former and current graduate students in Civil Engineering, for their contributions to the study.

2. Loadings

The loadings which are appropriate for a very stiff structure buried in a well-compacted fill may be the subject of a geotechnical investigation or of a soil-structure interaction analysis, but they are beyond the scope of this report.

The loadings which were considered in the study were those currently being specified by the Corps of Engineers. Two separate loading combinations are considered, and each members is proportioned for the most severe combination of forces. The basic loading is the case of the vertical pressure being 1.5 times the overburden in combination with a horizontal pressure of 0.5 times the overburden. The second loading condition is that of both vertical and horizontal pressures equal to the overburden. Both cases are illustrated in Fig. 2 for an overburden of 30 k/ft^2, and the loads are assumed to be uniformly distributed. The basic loading will govern the design of the horizontal and interior vertical members. The second loading will normally control the design of the outer vertical members.

The loads, and especially the horizontal loads, must be carefully evaluated. The flexural and shear strengths of the members are greatly enhanced by the presence of axial compression. Consequently, when designing the horizontal members, it is conservative to make a low estimate of the horizontal load, and it is decidedly nonconservative to overestimate the horizontal load. Similarly, it is nonconservative to overestimate the vertical load while designing the exterior vertical members to resist horizontal loads.

3. Idealization of Structure

The idealization of the structure into the mathematical model to be analyzed is a problem that the structural engineer faces daily, when applying elastic frame analysis techniques to reinforced concrete structures.

In all analyses of reinforced concrete structures, some approximations have to be made in determining the variations in member rigidity, or EI, along the lengths of members. The most usual assumption, for the part of the analysis concerned with finding the applied moments, shears, and axial forces, is that the variation in the gross concrete section will be considered. Thus, constant depth members usually are idealized as prismatic, even though there may be substantial variation in the reinforcement and in the extent of cracking along the members.

In frames of normal proportions, the next assumption will probably be that the frame is idealized as a line structure, and the possible effects of material in the joints on member stiffness are ignored. This leads to the familiar rotational stiffness expression of $K = 4EI/\ell$, for a member which is fixed at the far end and subjected to a moment at the simply supported end.

However, as the widths of the supporting members become larger relative to the span, it becomes necessary to take these widths into account if the distribution of moments is to be adequately predicted. As the supports become wider, one must also reconsider the definition of the span (clear span or center-to-center span, for example) and just what effects the growing joint areas have on the flexural stiffness factors.

In the earlier study (6), it was recommended, for instance, that in members without haunches, the flexural stiffness of each member be evaluated assuming the entire connection area of the member to be rigid. This results in higher member stiffnesses, larger carry-over factors, and higher fixed end moments than if the member is assumed prismatic for its entire length. After finding the balanced joint moments in the frame, the design moments were taken as the negative moments at the faces of the supports (and not at the centers of the supports) and as the maximum positive moments in the spans. Reference 6 also contains recommendations and discussion of cases with haunches.

The problem of the assumptions about the variation of moment of inertia near the ends of the members was discussed in Ref. 3. The results of several elastic frame analyses and of a number of finite element analyses were compared. It was concluded that the frame analysis method gave moments in reasonable agreement with the more complex analyses and with the test results if half the joint length was assumed rigid. The joint lengths included the lengths of the small fillets used in the test specimens. The rigid length is illustrated in Fig. 3.

The critical section for moment at the ends of the members has been taken at the tips of the fillets, as is also shown in Fig. 3. This location, rather than the face of the supporting member, was used as a result of studies of the test results and of a series of trial calculations of design moments.

The idealization of a particular structure is illustrated in Fig. 4. The structure is the prototype from which the dimensions of test specimens

R2 through R8 were scaled (1,3). The line-element structure has spans equal to the distances between centers of supporting members.

There are small cantilevers at each corner of the structure. These are included so that the idealized structure has the same total length and height as the real structure, and consequently has the same total load. Without these members, only 25.5/27.5 of the distributed vertical load on the real structure would be accounted for, and only 11/13 of the horizontal load would be included. Presumably nearly all of the load on a horizontal cantilever is resisted by the end vertical member, and the load on the cantilever could also be applied as an axial concentrated load without changing the overall results very much.

The rigid lengths in this case are 9 in. at each end of each member. The joint length in each case is the 12 in. half-width of the perpendicular member plus the 6 in. fillet, for a total of 18 in. Half of this is taken as the equivalent rigid length.

The three-dimensional conduit is also normally considered as a series of two-dimensional frames, or "slices", cut from the conduit. Various slices may have different loads as the overburden changes along the axis of the conduit, or through the thickness of the dam or fill. This is a reasonable assumption in most cases, and especially in view of the current Corps of Engineers practice in placing water-stopped joints which have no tensile capacity at relatively small intervals along the length of the conduit. If the conduit were built monolithically, this design procedure would still be adequate for the individual cross sections as illustrated in Fig. 4(a). There would be an entirely different analysis and design problem for forces acting parallel to the axis of the conduits. In the example that is considered in this report, a 12 in. length of the conduit will be analyzed as if it were an isolated from 12 in. wide.

4. Analysis of Structure

Once the structure has been idealized as was discussed in the previous section, the next step is the analysis to determine the controlling moments, shears, and thrusts at each critical section.

With the current availability of general frame analysis programs, this presents no difficulty. The idealized structure is described, the loadings are given, and the program is instructed to give moments at or near midspan and at the sections at the tips of the fillets, shears at the ends of the members or other sections, and the axial forces in each member. The STRUDL program was used in the study in Ref. 3.

The following material is specifically for the case of hand calculations, and is presented in a moment-distribution format, with and without various shortcuts.

The nonprismatic members introduce some complexities, but the three curves shown in Fig. 5 minimize these. Figure 5, plotted from data in Ref. 7, gives Fixed-End Moments, Carry-Over Factors, and Flexural Stiffness Factors for members with rigid areas near each end. This is for the specific case of rigid areas of the same length at both ends of the member. Coefficients for cases with different rigid lengths at the two ends of a member may be found using basic principles of analysis or through the use of computational devices such as the Column Analogy (8). (If the rigid lengths are nearly the same, reasonable approximations can be obtained by assuming both ends to have the shorter rigid length when considering the end with the shorter rigid length, and both to have the longer rigid length when considering the end with the longer rigid length.)

Once these factors have been determined, the moments are distributed until a satisfactory balanced condition has been obtained. The design forces are then determined, starting from the joint forces.

To illustrate this process, the structure shown in Fig. 4 will be analyzed, assuming a vertical load of 45 k/ft^2 and a horizontal load of 15 k/ft^2. Some results of the analysis with both vertical and horizontal loads equal to 30 k/ft^2 will also be presented. There are only two kinds of members, so the determination of the coefficients is relatively simple. The joints have been identified in Fig. 4(b), to aid in the bookkeeping operation.

For the horizontal members:

$$\ell = 8'\text{-}6" = 102 \text{ in.}; \quad a\ell = 9 \text{ in.}; \quad a = 0.0882$$

From Fig. 5: $\text{FEM} = 0.0967\, w\ell^2$

$$\text{COF} = 0.633$$

$$K = 6.6EI/\ell = 6.6EI/8.5 = 0.776\ EI$$

For the vertical members:

$$\ell = 11'\text{-}0" = 132 \text{ in.}; \quad a\ell = 9 \text{ in.}; \quad a = 0.0682$$

From Fig. 5: $\text{FEM} = 0.0940\, w\ell^2$

$$\text{COF} = 0.604$$

$$K = 5.8EI/\ell = 5.8EI/11 = 0.527\ EI$$

All members have the same thicknesses in this case, so the EI values are the same. If a "slice" of the conduit 12 in. wide is considered, the load per sq ft is also the load per ft of length of member, and the resultant moments, shears, and thrusts are per ft of width of structure.

Using the above information, the fixed end moments become:

Horizontal members: FEM = $0.0968\ (45\ \text{k/ft})(8.5\ \text{ft})^2 = 314.4$ k-ft

Exterior vertical members:

$$\text{FEM} = 0.0940\ (15\ \text{k/ft})(11\ \text{ft})^2 = 170.6\ \text{k-ft}$$

Interior vertical members: FEM = 0

Horizontal cantilever: $M = 45\ \text{k/ft}\ (1\ \text{ft})^2/2 = 22.5$ k-ft

Vertical cantilever: $M = 15\ \text{k/ft}\ (1\ \text{ft})^2/2 = 7.5$ k-ft

The distribution factors at the corner and interior joints are determined:

Joint A	Relative values = Distribution Factors
K_{AB} = 0.776 EI	0.596
K_{AE} = 0.527 EI	0.404
ΣK = 1.303 EI	1.000

Joint B	Relative values = Distribution Factors
K_{BA} = 0.776 EI	0.373
K_{BC} = 0.776 EI	0.373
K_{BF} = 0.527 EI	0.254
ΣK = 2.079 EI	1.000

This information can then be assembled into a traditional moment-distribution format, as shown in Fig. 6. Because of symmetry, only one quarter of the structure need be considered, with leads to considerable reduction in arithmetic. Rotational signs, with clockwise moments on the end of a member as positive, are used. The two cantilever moments are added together and treated as an applied moment, and of course there is no distribution or carry-over to these members. The carry-overs for

the vertical members and the central horizontal span are from symmetrical portions of the structure which are not shown. They can also be viewed as negative reflections of the carry-overs from the joints actually being considered. Convergence is relatively slow because of the large carry-over factors, but the 5 cycles shown are more than adequate.

Since the vertical and interior horizontal members are subjected to the same moments, with opposite rotational signs, at the two ends, another short-cut can be introduced by using stiffness factors modified for this condition of symmetry. If this is done, the carry-overs along these particular members are eliminated, and some savings result in the analysis time.

The modified stiffness of a member with equal but opposite end moments can be expressed as (Ref. 9):

$$K' = K\ (1 - COF).$$

With this, the central horizontal member has a modified stiffness of

$$K' = \frac{6.6\ EI}{8.5}\ (1 - 0.633) = 0.285\ \ EI.$$

The vertical members have modified stiffnesses of

$$K' = \frac{5.8\ EI}{11}\ (1 - 0.604) = 0.209\ EI.$$

Using these modified stiffnesses, new distribution factors for the two joints can be found in the same way that the previous values were computed. The modified distribution factors are shown in Fig. 7, where a new moment distribution is shown. The small differences between the moments in Figs. 6 and 7 are due to accumulations of round-off errors and are insignificant.

Since these are joint moments, the next step is to determine the design moments, shears and thrusts. This can be illustrated graphically, and Fig. 8 shows the moments and shears in the end horizontal span.

The moment diagram in Fig. 8 shows the maximum span positive moment and the two negative moments at the tips of the fillets. The moment at the exterior corner of the structure is very small, and the second loading with 30 k/ft^2 in both horizontal and vertical directions would give a much larger negative moment, of about 127 k-ft. This small moment is consistent with the results of the elastic finite element analysis presented in Ref. 3.

The moments at all critical sections would then be determined, as would the thrust accompanying each moment. The thrusts for the horizontal members can be found by simple statics since the top and bottom members each resist half the horizontal load. The vertical member thrusts are found by summing reactions at the ends of the horizontal members, not forgetting the reactions from the small corner cantilevers.

The moments and thrusts at each critical section are summarized in Fig. 9 for the two loading cases. The end span positive moments are the maximums in the span rather than the slightly smaller midspan values.

At some locations, such as at the ends of the exterior vertical members, it may be necessary to consider two combinations of moment and thrust, as it may not be evident which loading governs. In addition, the two different loadings produce moment diagrams with different points of contraflexure, and this will have to be taken into account when detailing the reinforcement.

The critical section for shear has been taken at 0.15 ℓ_n from the face of the supporting members, where ℓ_n is the clear span between the supports, neglecting the presence of the fillets. This is an arbitrary definition which was developed from the analysis of the test results reported in Ref. 3, and would not necessarily be reasonable if much larger corner fillets were used. The fillets obviously influenced the paths of the shear cracks in the test specimens, but their size was not a variable in the test program.

The shears at the various critical sections, from the two loadings, are shown in Fig. 9.

Please note that the forces shown in Fig. 9 are service load forces, and they must be multiplied by an appropriate factor of safety before proportioning the cross sections using an ultimate strength design approach. The remainder of this report is written on the basis that 2.0 is a reasonable factor, and that it includes both the overload and under strength components of the factors of safety as used in the ACI Code (10). This factor is applied to all members resisting all combinations of forces.

5. Proportioning of Cross Sections

The first step in checking the adequacy of the structure which has just been analyzed should be the confirmation of the shear strength. It has been assumed that no shear reinforcement will be used, so the concrete section must be adequate. The test results indicated no more than a weak dependence of the shear strength on the flexural steel ratio, and there appears to be no difficulty in providing flexural capacity to match or exceed the shear capacity.

The minimum shear capacity of a critical section can be computed, following the recommendations of Ref. 3, as:

$$v_c = (11.5 - \ell_n/d)\sqrt{f'_c}\,\sqrt{1 + f_h/f_t} \tag{5-1}$$

where

$v_c = V_c/bd$

V_c = Shear capacity of section of width b,

d = Effective depth of reinforcement, from compression face,

ℓ_n = Clear span face to face of supporting members, neglecting fillets,

f'_c = Compressive strength of concrete, lb/in.^{2}, ($\sqrt{f'_c}$ also has lb/in.^{2} units),

$f_h = N_u/A_g$ = Nominal longitudinal stress in member,

f_t = Tensile strength of concrete, taken as $5\sqrt{f'_c}$,

N_u = Axial thrust in member, compression positive, and

A_g = Gross area of member.

The critical section is located 0.15 ℓ_n from the face of the supporting member, as noted in the previous section.

The cross section of each member will be approximately as shown in Fig. 10. The cover over the main reinforcement is 4 in., and is consistent with current Corps of Engineers practice for hydraulic structures (4). The transverse bars are nominal reinforcement placed perpendicular to the main steel. These bars provide crack control and some protection against deep erosion of the concrete by any cavitation problems, and are placed on the sides of members exposed to flowing water. The figure is drawn as if the main steel were #11 bars, but this of course remains to be determined.

Two sections need to be checked for shear. The interior negative moment section of the end horizontal span has maximum shear and minimum thrust when the vertical load is 45 k/ft^2. The exterior vertical member has the worst conditions when vertical and horizontal loads are both 30 k/ft^2.

Adopting a load factor of 2.0, the horizontal member is subjected to ultimate loads of V_u = 234.4 kips and P_u = 195.0 kips. The concrete compressive strength is taken as f'_c = 4,000 lb/in.^{2} Assuming #11 bars, d = 24 - 4 - 1.41/2 = 19.3 in. The clear span is ℓ_n = 78 in. The average longitudinal stress is f_h = 195 kips/(12 x 24) in.^{2} = 0.677 k/in.^{2}. The $\sqrt{f'_c} = \sqrt{4000}$ = 63.2 lb/in.^{2}, and $f_t = 5\sqrt{f'_c}$ = 316 lb/in.^{2}.

The applied shear stress is $v_u = V_u/bd$ = 234.4 kips/(12 x 19.3) in.^{2} = 1.01 k/in.^{2}.

The shear resistance is

$$v_c = (11.5 - 78/19.3)\ 63.2\ \text{lb/in.}^2 \sqrt{1 + 677/316} = 836\ \text{lb/in.}^2$$

The 836 lb/in.^{2} computed shear capacity is less than the desired 1010 lb/in.^{2}, and leads to a computed failure load of 74.5 k/ft.^{2} The designer has two choices if the structure is to support the 90 k/ft^2. The members may be made deeper, or the concrete strength may be made stronger, or both. Increasing the concrete strength alone would require f'_c = 7,000 lb/in.^{2}, so this is not practical as the only change. The shear strength increases at a rate less than $\sqrt{f'_c}$ because of the f_t term under the radical. A member depth of 28 in., 4 in. thicker, would result in the strength exceeding the factored applied load.

When compared directly with the results of the model tests, it is apparent that Eq. 5-1 is relatively conservative. The lowest failure load for a model of this structure under this loading configuration was 105 k/ft^2, with concrete only slightly stronger than 4,000 lb/in.^{2} On the basis of direct comparisons of the 105 k/ft^2 load from the tests of Models R6 and R8 (3) and half of the 74.5 k/ft^2 computed earlier, the factor of safety against shear failure appears to be about 2.8 rather than 2.0. Or, the measured failure load was about 40 percent higher than the computed failure load.

This is not entirely unreasonable, in view of the scatter in the shear strength data. The 40 percent excess is a result of Eq. 5-1 being a lower bound to all of the test data, including cases in which f_h, the horizontal compression, was zero.

The redesign will not be pursued, but rather the exterior vertical will be checked for shear, and then the flexural analysis considered.

In the exterior vertical member, the ultimate shear is $V_u = 189.0$ kips, in conjunction with a thrust of $N_u = 345.2$ kips, from the $w_v = w_h = 30\ k/ft^2$ loading, times 2.0. The nominal applied shear stress is $v_u = 816\ lb/in.^2$, and $f_h = 1,200\ lb/in.^2$ The shear resistance can then be computed as:

$$v_c = (11.5 - 108/19.3)\ 63.2\ lb/in.^2 \sqrt{1 + 1200/316} = 818\ lb/in.^2$$

The end vertical is thus adequate for shear, and would not have to be thickened in a redesign. The shear force would not be changed by additional depth in the horizontal members, and there could be no more than an extremely small change in the axial thrust.

On the basis of a direct comparison with the test of Model R7, the computed and measured shear strengths are the same. The other models were somewhat stronger.

The basis for Eq. 5-1 is explained in Ref. 3, and Fig. 11 is reproduced from that report to show the relative values of the test results and Eq. 5-1.

The proportioning of sections for flexure must begin with a judgement decision about the minimum amount of steel to be used, as this will often govern. It is suggested that a minimum area of 0.01 (1%) of the gross area of the section be adopted, and this be divided equally between the two faces of the member. For the section shown in Fig. 10, this results in a total area of steel of 2.88 $in.^2/ft$, and a tension steel ratio of 0.0062, where $\rho = A_s/bd$ and A_s = steel area in one face of the member.

The 1971 ACI Code (10) specifies a minimum tension steel ratio in a beam of $\rho_{min} = 200/f_y = 0.0033$ for grade 60 steel. However, this may not give adequate crack control, because the bar spacing can become quite large and because of the large cover. Information on the parameters influencing crack spacing and width can be found in Ref. 11, and Ref. 12 confirms that the same concepts can be extended to structures reinforced with very large bars.

The ACI Code also specifies a minimum reinforcement of 0.01 of the gross area of a compression member, and all members in the conduits are subjected to significant compression in addition to the bending forces. The area of 2.88 in.^{2}/ft can be met by #11 bars at 13 in. in each face, or by #8 bars at 6.5 in., or by other combinations. The #8 bars would be better for crack control purposes because of the smaller spacing, but the advantage may not be large.

Each critical section is subjected to some combination of moment and thrust, and it must be confirmed that the sections are adequate. A moment-thrust interaction diagram was consequently developed for the cross section shown in Fig. 10, and is shown in Fig. 12. The diagram is specifically for the case 1.44 in.^{2}/ft of steel in each face of the member, d = 19.3 in., f'_c = 4,000 lb/in.^{2}, and f_y = 60,000 lb/in.^{2}

The method of determining the moment-thrust interaction diagram is documented in standard text books (13, 14) on reinforced concrete, and will be only briefly reviewed here. Figure 13 is reproduced from Ref. 3, and illustrates the strain, stress, and force conditions associated with one point on this interaction envelope. In essence, a strain distribution such as is illustrated is selected, forces are found, and forces and moments of forces are summed about the mid-depth axis. Some other strain

distribution is then selected, and the process is repeated, with enough trials to define the envelope to the required precision.

The strain distribution producing the failure strain in concrete and the yield strain in the tension steel leads to a balanced condition, and the moment and thrust define a major change in slope of the envelope.

The stress distribution within the concrete is the simple approximation suggested by the ACI Code. It will be adequate except in the region of very high thrust where the entire cross section is in compression and the neutral axis lies outside the section. Consequently, kd = h should be the largest kd value chosen using the stress conditions shown in Fig. 13. This point can then be connected by a straight line with a point representing the pure axial thrust capacity, given by

$$P_o = (A_s + A_s') f_y + (A_g - A_s - A_s') 0.85 f_c' \qquad (5\text{-}2)$$

This force exists with zero moment if the section is symmetrical about the mid-depth axis. Otherwise, the moments of the forces must be summed about that axis. References 13 and 14 can be used to obtain further guidance in nonsymmetrical cases.

The curve shown in Fig. 12 does not have the shape normally shown for interaction diagrams. The portion below the balance point is straighter than is usually encountered, and the portion above the balance thrust has much more curvature than is normal. These changes in shape are due to the large cover over the compression reinforcement, combined with the use of Grade 60 steel. As a consequence of these two factors, the strain in the compression steel remains less than the yield strain

even at the balance point failure conditions, and in this particular case for any thrust less than 525 kips even when the moment is increased to failure. The normal shape illustrated in the text books exists only if the compression steel has yielded at the balanced failure condition, and for somewhat lower thrusts as well.

Once the envelope was prepared, the applied moment and thrusts were plotted for several of the critical sections. The rays from the origin represent "load paths" showing the combinations of M and P as the loads are increased from zero to twice the design values. Only the positive moment in the end span horizontal member (which will have to be thickened for shear) and the midheight section of the exterior vertical member are even close to the envelope. The other sections have considerable excess capacity, even with the minimum reinforcement.

The interior vertical member is subjected to only a very small moment and a large thrust, and these have been plotted. A ray and point are also plotted corresponding to a minimum thrust eccentricity of 0.1 times the member thickness, or 2.4 in., as is required by the ACI Code for columns. This minimum eccentricity is a reasonable requirement, and it can be argued that it is essential to protect the structure from small accidental moments arising from accidental nonsymmetries of the loads and of the structure.

6. Detailing of Steel

Once the required areas of steel have been found for all of the critical sections, the arrangement of this steel in the structure must be completed. The lengths of bars, the locations of bar terminations, and the locations of bends must be determined, and this should be done to both insure the adequate behavior of the structure and to minimize material and fabrication costs.

A simple steel arrangement was used in the models, and the results were satisfactory. The steel used is shown in Fig. 14, which is a drawing of a model 1/3 the size of the prototype discussed earlier in this report.

The outside steel is bent around the corners in the model, and the verticals are then lap-spliced in a region of generally compressive stress. All other bars are straight and full length. No bars are cut off short, even though in some cases the shape of the bending moment diagrams might appear to allow this to be done.

The extensions of the inside bars (bottom bars in top member, top bars in bottom member, and inside bars in exterior vertical members) into and nearly through the supports is critical.

In the test specimens, shear cracks formed near the ends of the members, and approximately lined up with the fillets, as shown in Fig. 15. The formation of these cracks was accompanied or preceeded by a change from compression to tension strains in the reinforcing bars near the tips of the fillets. The tension in these nominally compression bars became reasonably large in some cases, and could have lead to bond failures and other problems if the embedments of the bars beyond this section had been smaller, or if some bars had been cut off before this section was reached.

These reversals from compression to tension were noted both at exterior corners and at interior joints, which is the reason for the recommendation that the bars be full-length.

The use of all straight bars rather than bent bars is recommended. Bars of the form shown in Fig. 16 have been used to supply about half of the steel in some conduit structures, but these become quite inefficient and ineffective when members with small ℓ_n/d values are considered. If the sloping parts of the bars are to be no steeper than 45° from the member axis, the straight horizontal lengths available at either face of the member become small, and too much of the bar is in the sloping parts. Only about half of the span shown in Fig. 16 is reinforced by the straight, horizontal parts of the bar.

The use of two different loadings which cause large shifts in the points of contraflexure tend to make selection of efficient bent steel even more difficult.

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Note:

Construction Joints and Piping Locations not shown.

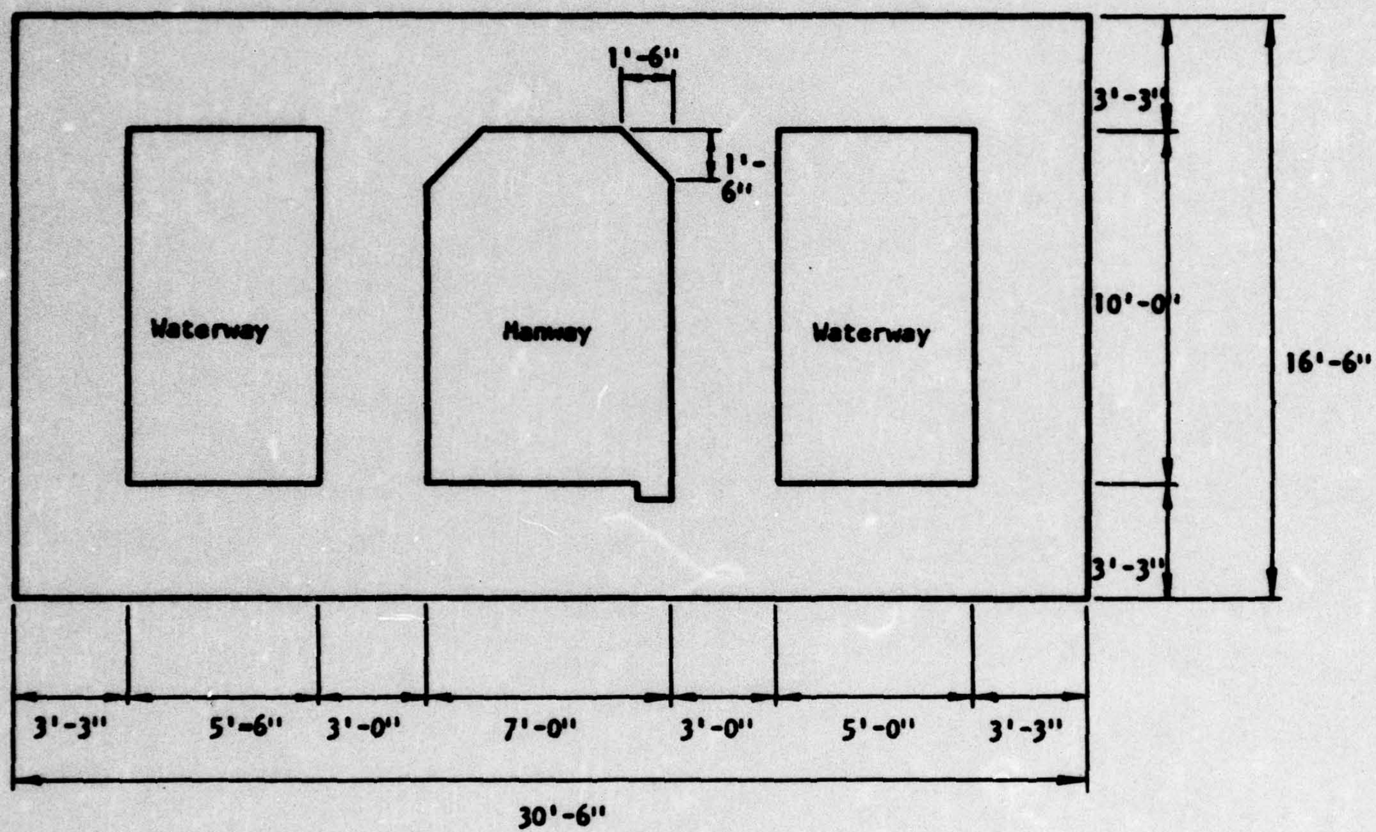


FIG. 1 CROSS-SECTION OF CONDUIT THROUGH DAM AT FALL CREEK RESERVOIR, OREGON

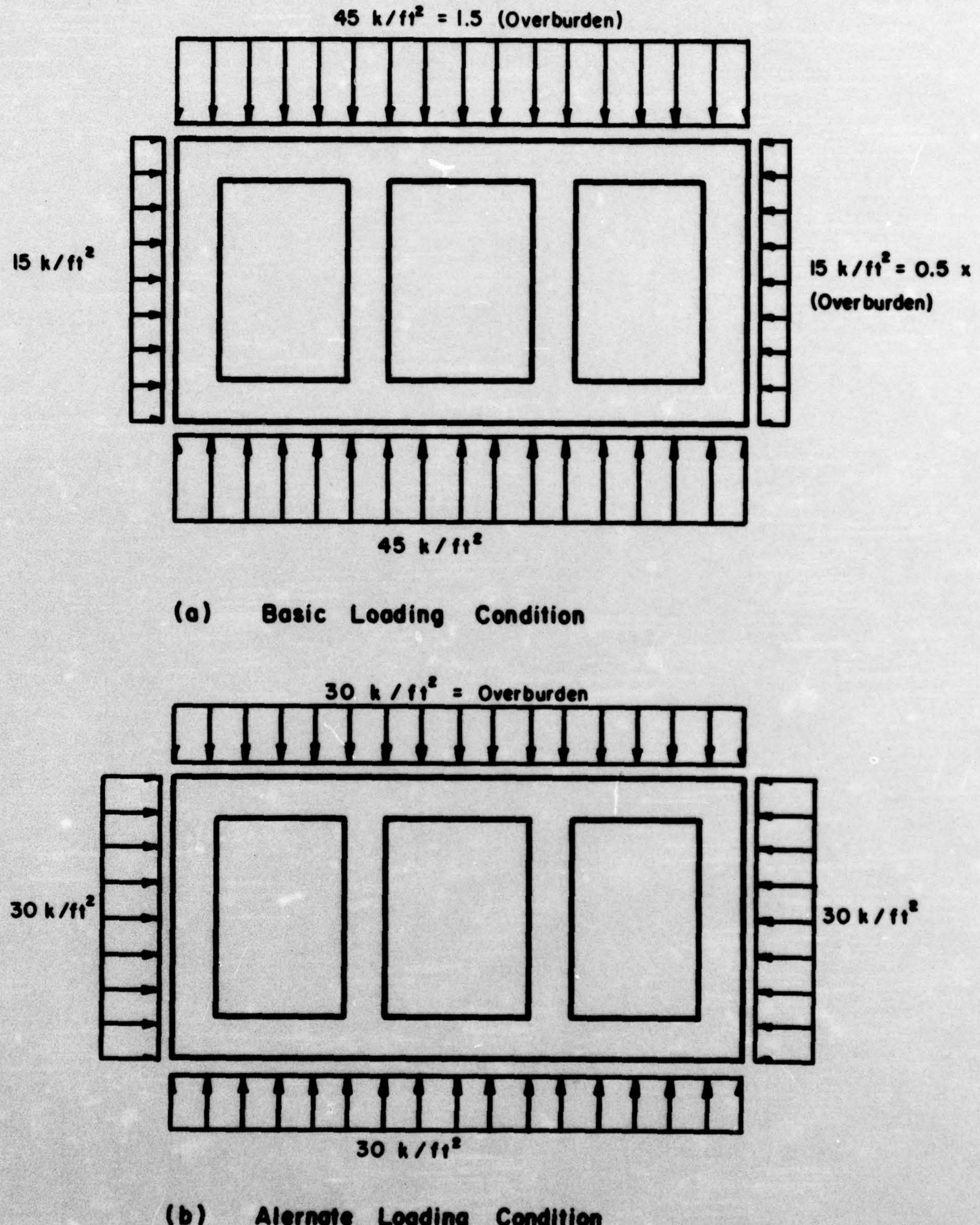


FIG. 2 LOADING CONDITIONS CONSIDERED IN DESIGN OF CONDUITS

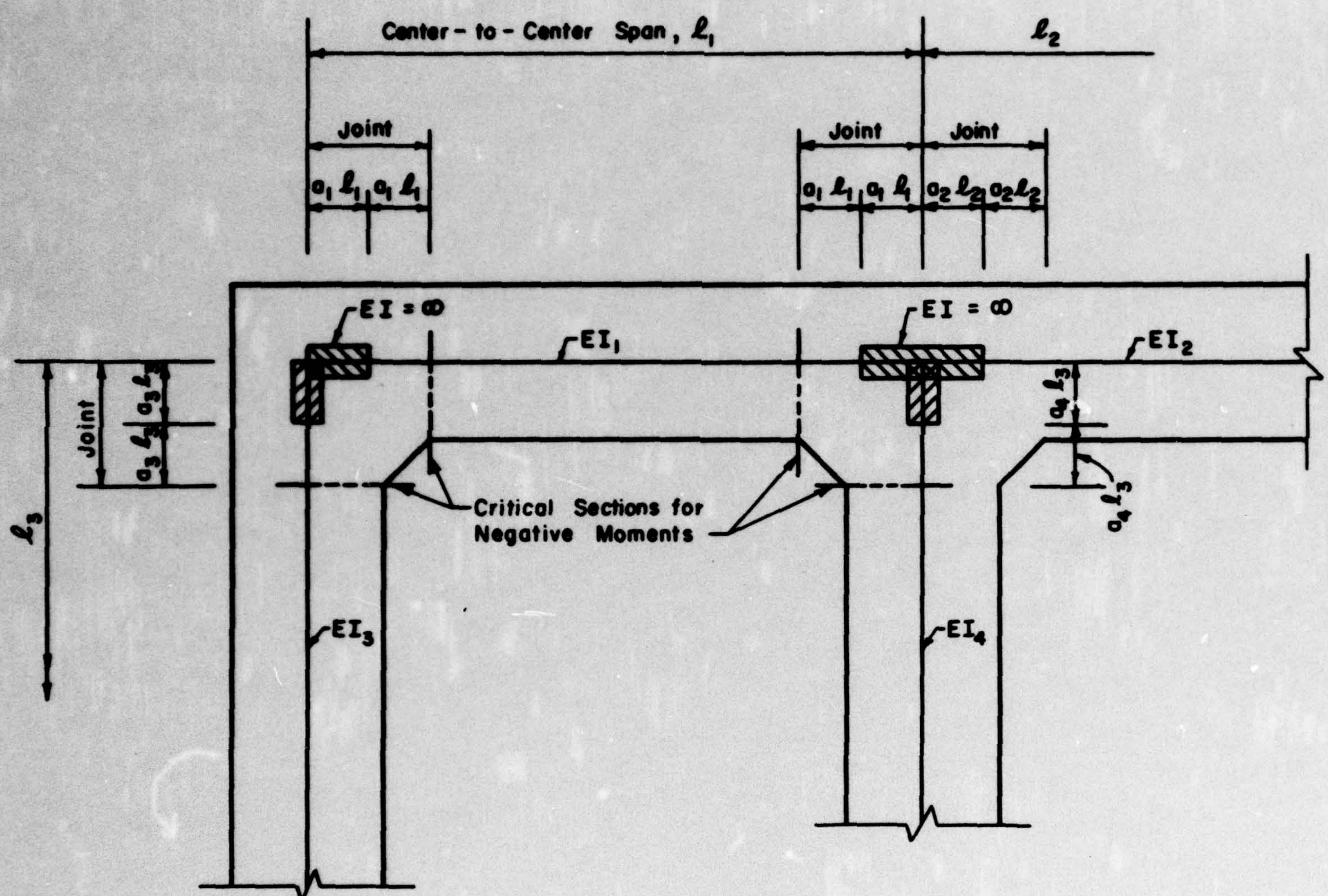
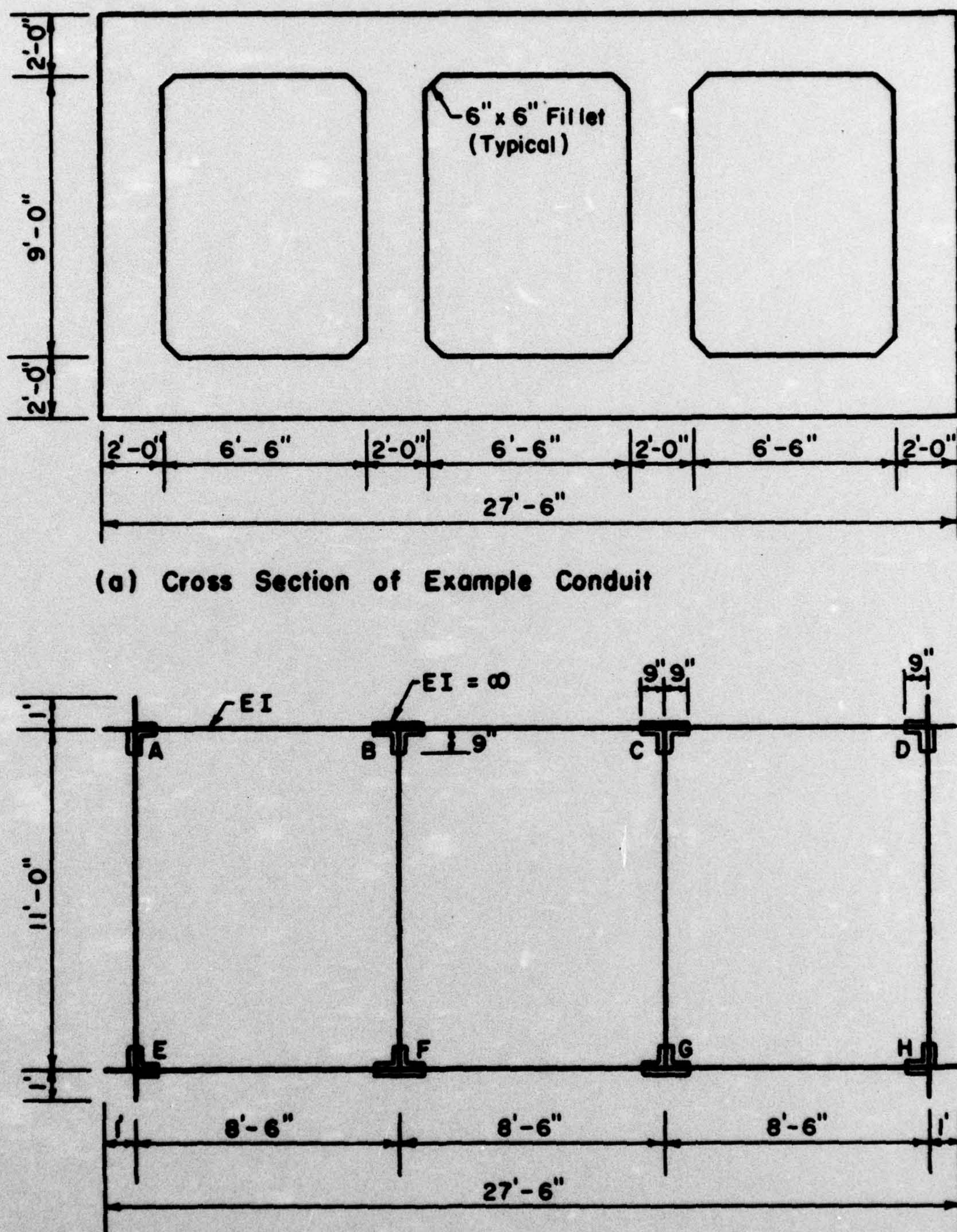


FIG. 3 DEFINITIONS OF RIGID LENGTHS AND LOCATIONS OF CRITICAL SECTIONS FOR NEGATIVE MOMENT



(a) Cross Section of Example Conduit

(b) Idealization of Conduit Into Frame

FIG. 4 EXAMPLE CONDUIT AND ITS IDEALIZATION AS A FRAME

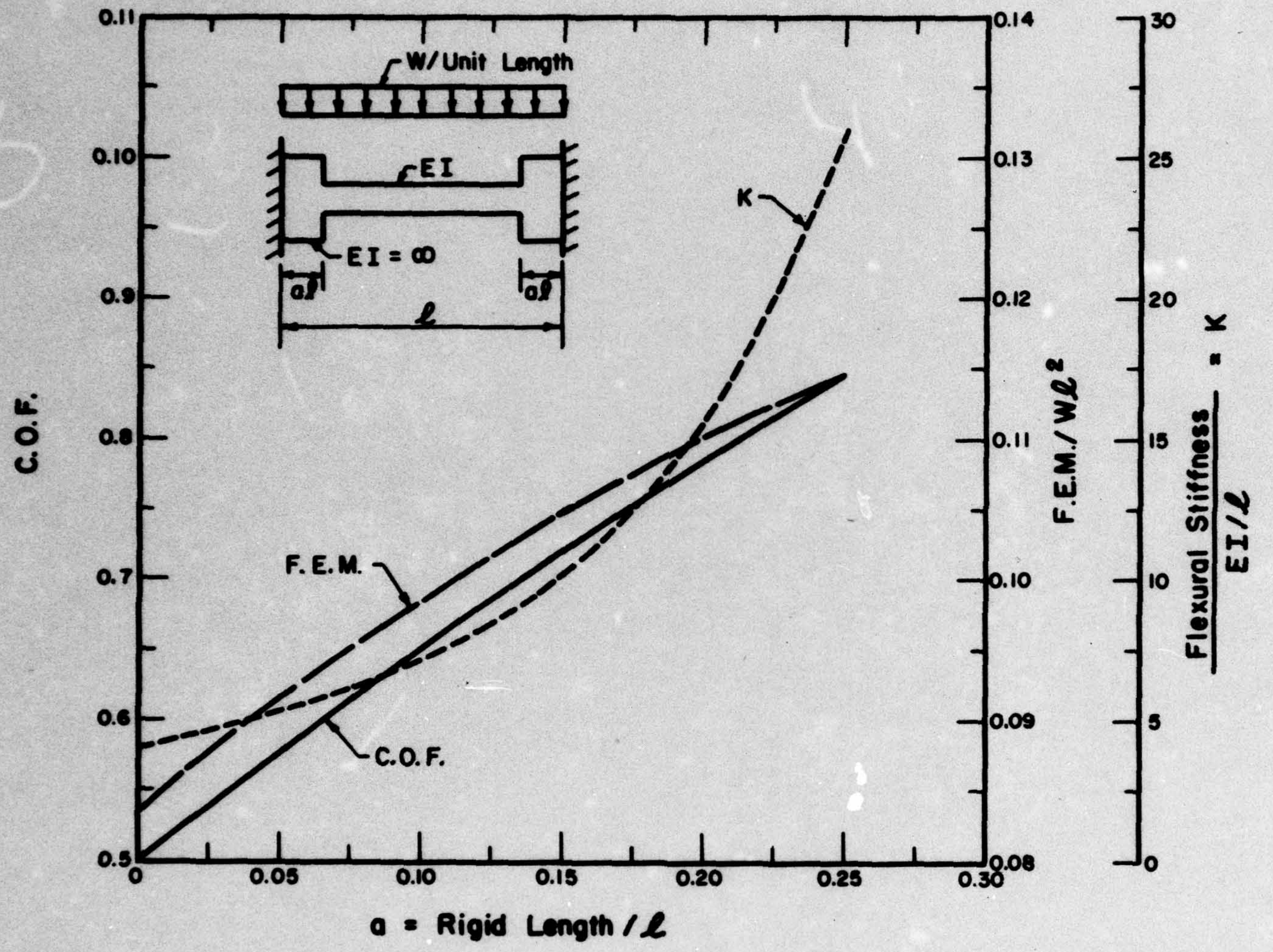


FIG. 5 STIFFNESS AND CARRY-OVER FACTORS AND FIXED-END MOMENTS AS FUNCTIONS OF RIGID LENGTHS AT MEMBER ENDS

$W_v = 45\ k/ft^2$ $W_h = 15\ k/ft^2$

− ⟲ ——— ⟳ +

A B (C)

	M_{corner}	M_{AE}	M_{AB}	M_{BA}	M_{BF}	M_{BC}
Distribution Factors	0	0.604 ↓	0.633 →	← 0.633	0.604 ↓	0.633 →
F.E.M. (k-ft)	+15	+170.6	-314.4	+314.4	0	-314.4
Bal.	0	+52.0	+76.8	0	0	0
Carry-Over	0	-31.4	0	+48.6	0	0
Bal.	0	+12.7	+18.7	-18.1	-12.4	-18.1
Carry-Over		-7.7	-11.5	+11.8	+7.5	+11.5
Bal.	0	+7.8	+11.4	-11.5	-7.8	-11.5
Carry-Over		-4.7	-7.3	+7.2	+4.7	+7.3
Bal.	0	+4.8	+7.2	-7.2	-4.8	-7.2
Carry-Over		-2.9	-4.6	+4.6	+2.9	+4.6
Bal. (Close Enough)		+3.0	+4.5	-4.5	-3.1	-4.5
Summation	+15.0	+204.2	-219.2	+345.3	-13.0	-332.3

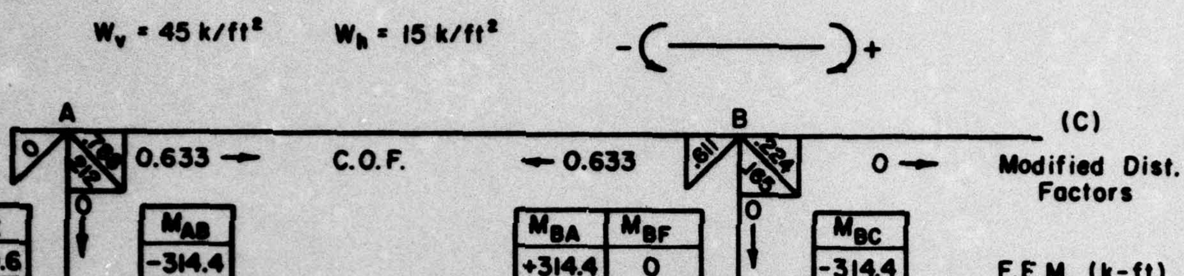
C.O.F.

.373 .373 .254

(E) (F)

FIG. 6 MOMENT DISTRIBUTION PROCESS FOR EXAMPLE CONDUIT, BASIC METHOD

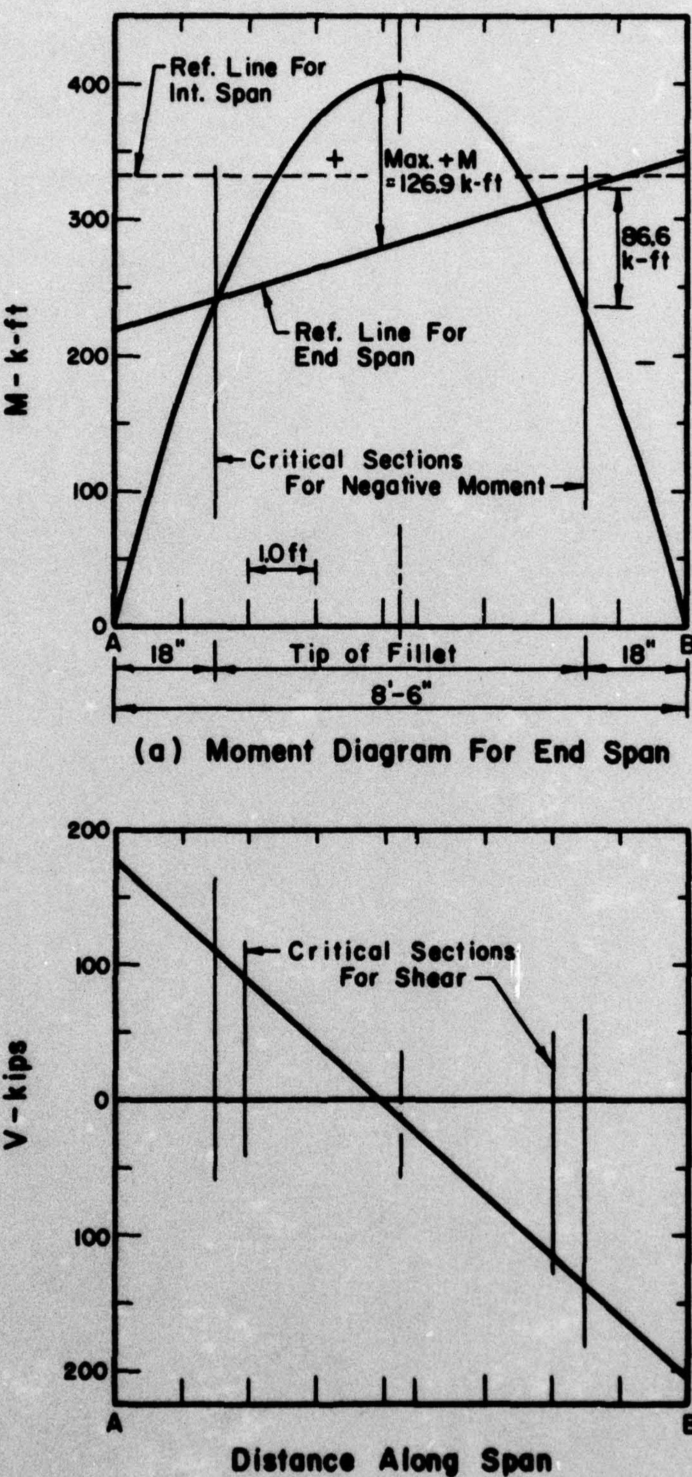
$W_v = 45\ k/ft^2$ $W_h = 15\ k/ft^2$



	M_{corner}	M_{AE}	M_{AB}	M_{BA}	M_{BF}	M_{BC}
F.E.M. (k-ft)	+15	+170.6	-314.4	+314.4	0	-314.4
Bal.	0	+27.3	+101.5	0	0	0
Carry-Over		0	0	+64.2	0	0
Bal.	0	0	0	-39.2	-10.6	-14.4
Carry-Over		0	-24.8	0	0	0
Bal.	0	+5.3	+19.5	0	0	0
Carry-Over		0	0	+12.3		
Bal.	0	0	0	-17.5	-2.0	-2.8
Carry-Over			-4.7	0	0	0
Bal.	0	+1.0	+3.7	0	0	0
Carry-Over				+2.3		
Bal.				-1.4	-0.4	-0.5
Summation	+15.0	+204.2	-219.2	+345.1	-13.0	-332.1

(E) (F)

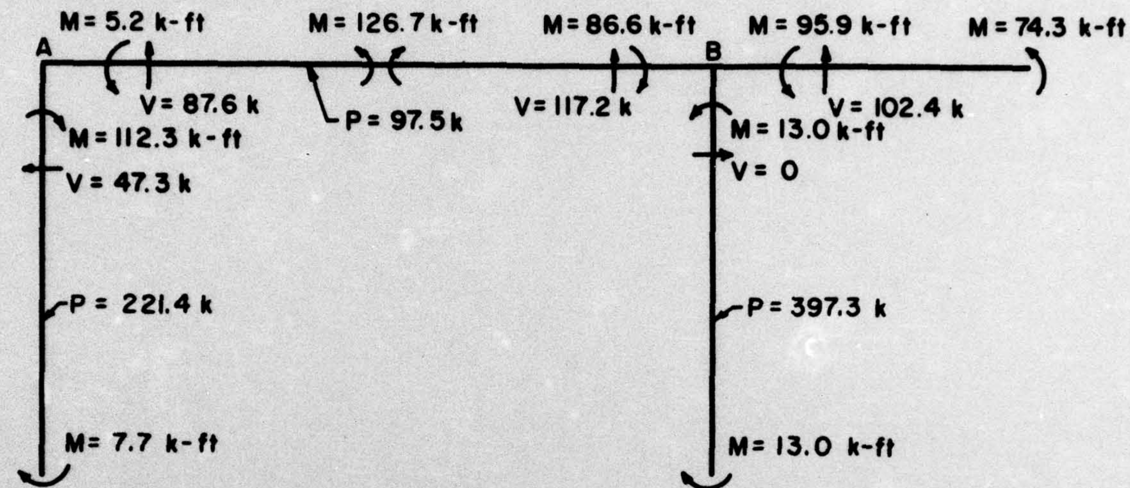
FIG. 7 MOMENT DISTRIBUTION PROCESS FOR CONDUIT, WITH MODIFIED STIFFNESS FACTORS



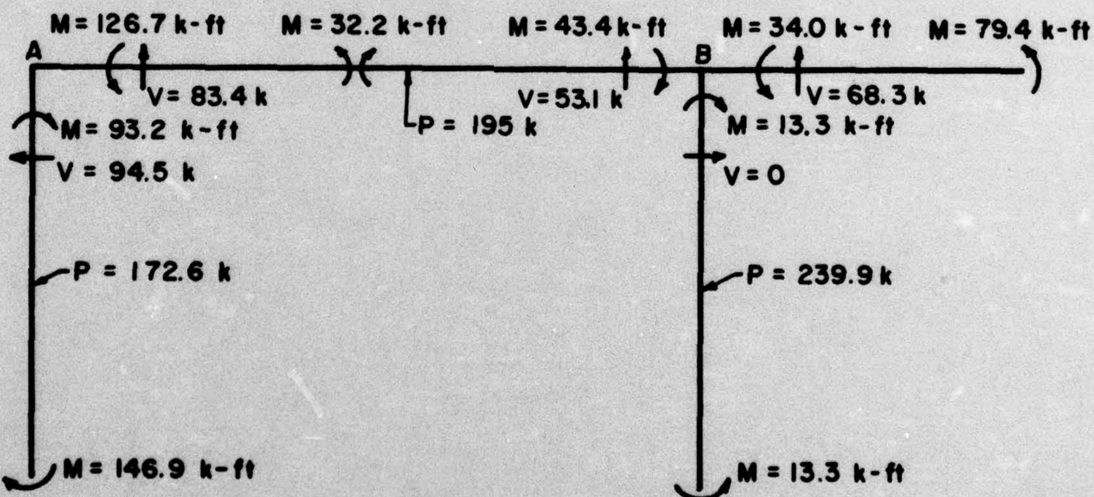
(a) Moment Diagram For End Span

(b) Shear Diagram For End Span

FIG. 8 GRAPHICAL REPRESENTATION OF DETERMINATION OF MOMENTS AND SHEARS AT CRITICAL SECTIONS



(a) $w_v = 45\ k/ft^2$, $w_h = 15\ k/ft^2$



(b) $w_v = w_h = 30\ k/ft^2$

FIG. 9 SUMMARIES OF MOMENTS, SHEARS, AND THRUSTS AT CRITICAL SECTIONS FROM THE TWO LOADINGS

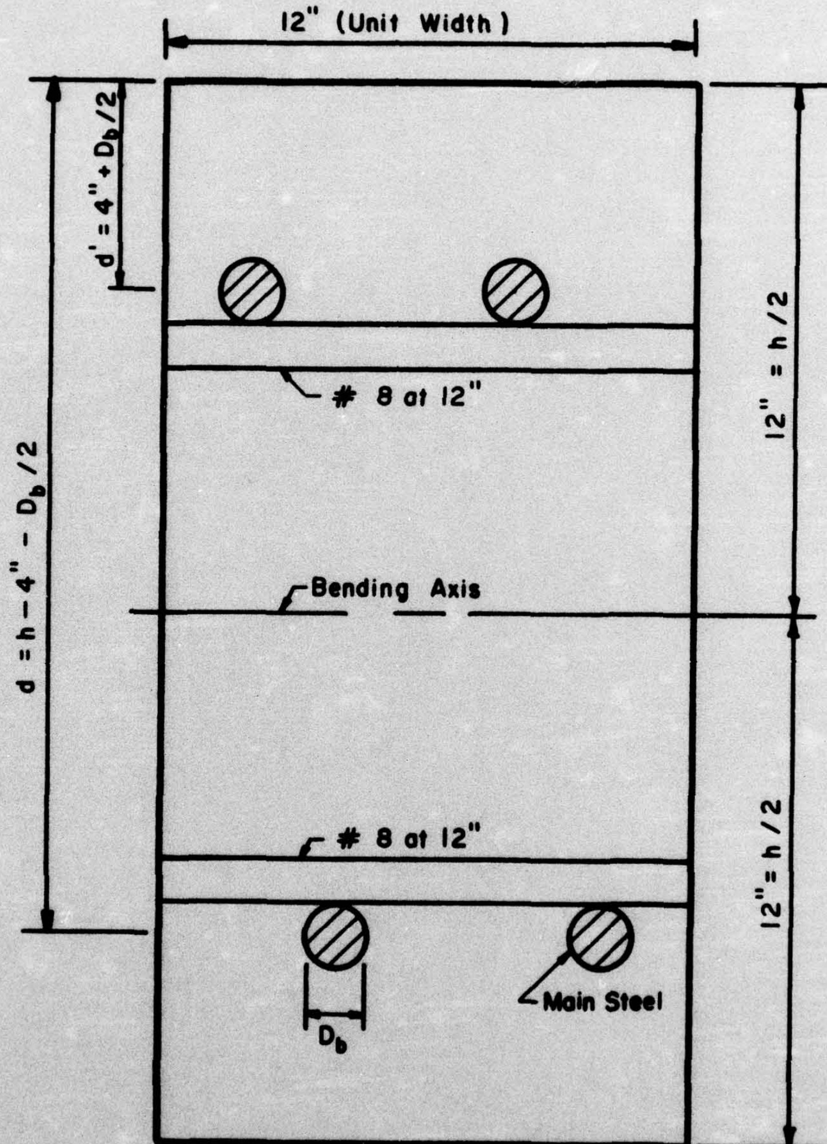


FIG. 10 CROSS SECTION OF UNIT WIDTH OF CONDUIT MEMBER

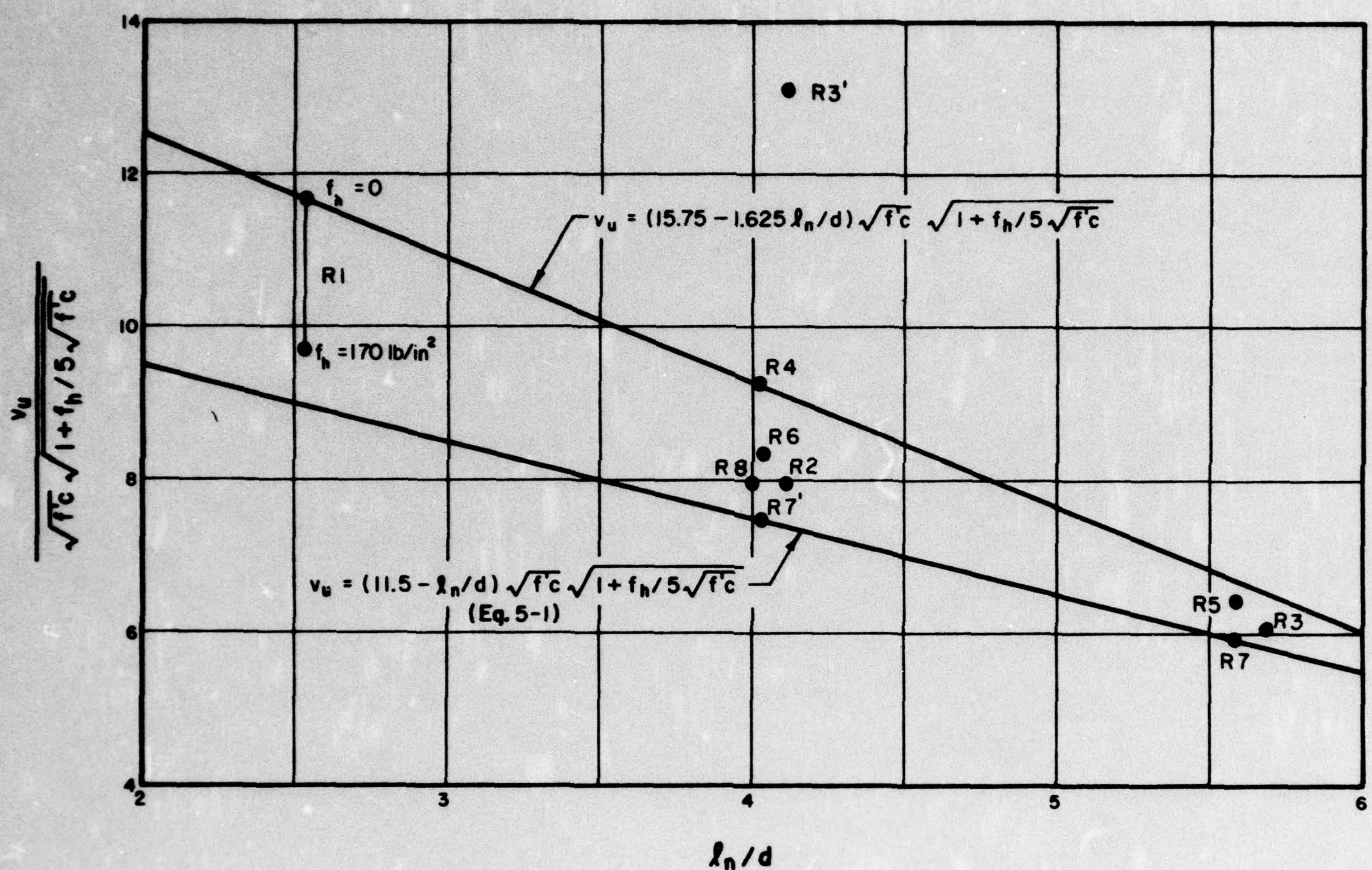


FIG. 11 NORMALIZED SHEAR STRENGTH DATA, INCLUDING THRUST EFFECTS, VERSUS SPAN-DEPTH RATIO

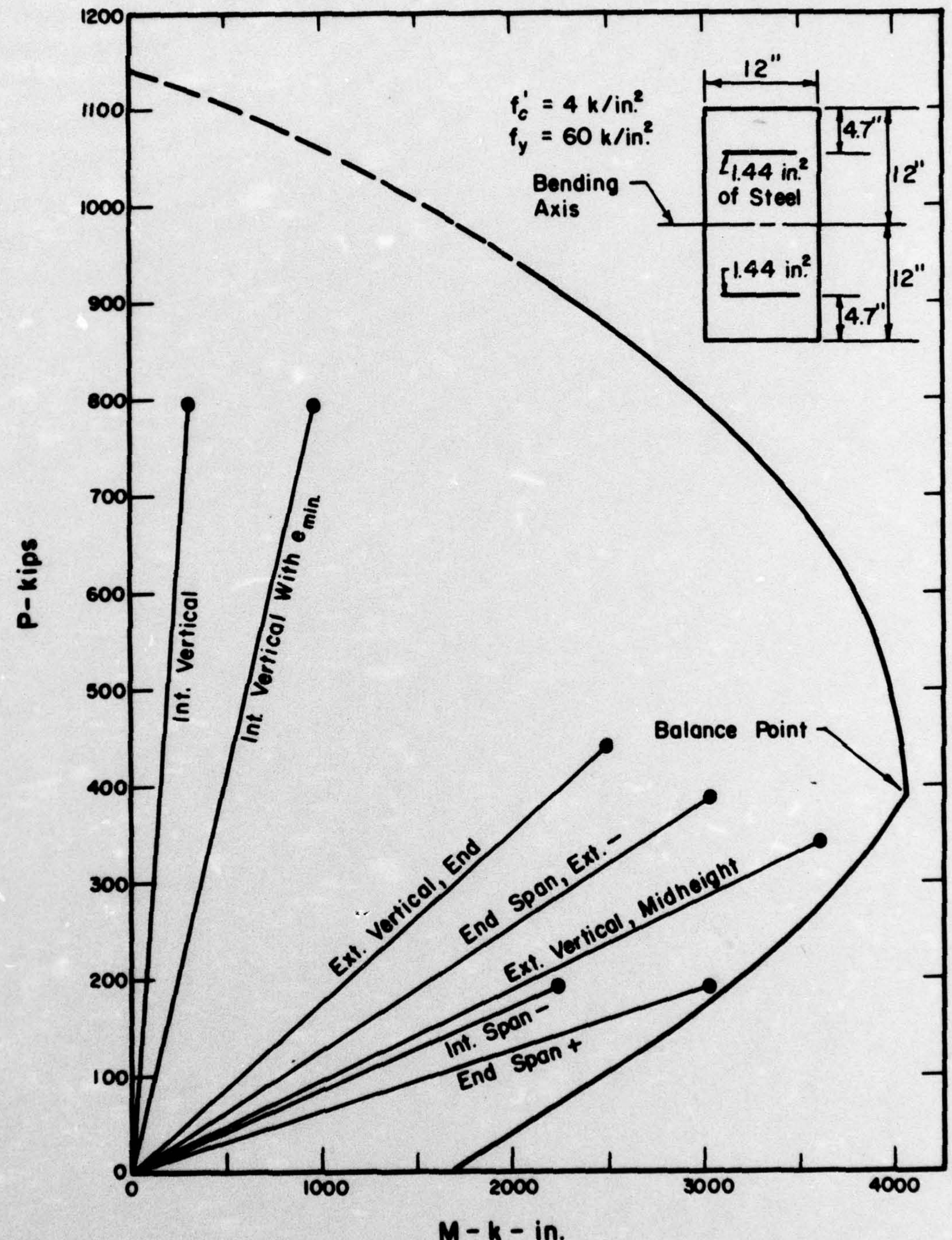
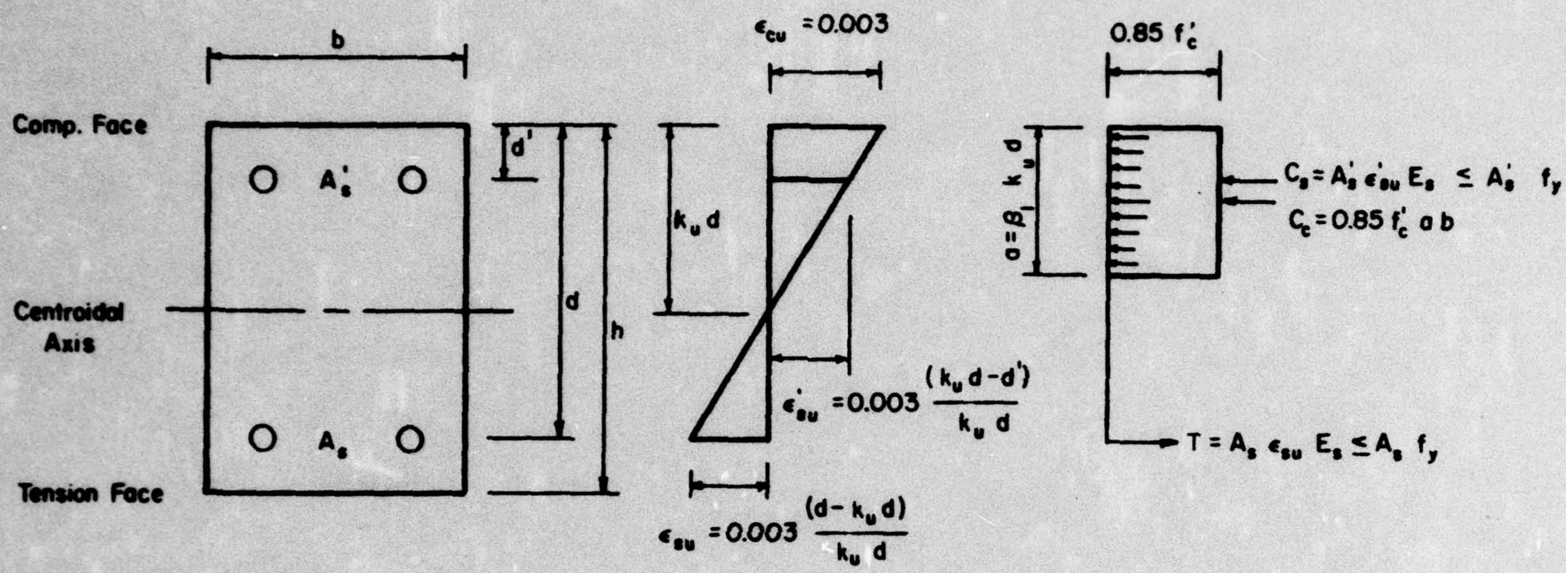


FIG. 12 MOMENT-THRUST INTERACTION DIAGRAM FOR CONDUIT SECTION AND APPLIED MOMENTS AND THRUSTS



Section $(A'_s = A_s)$ — Strains — Stresses And Forces

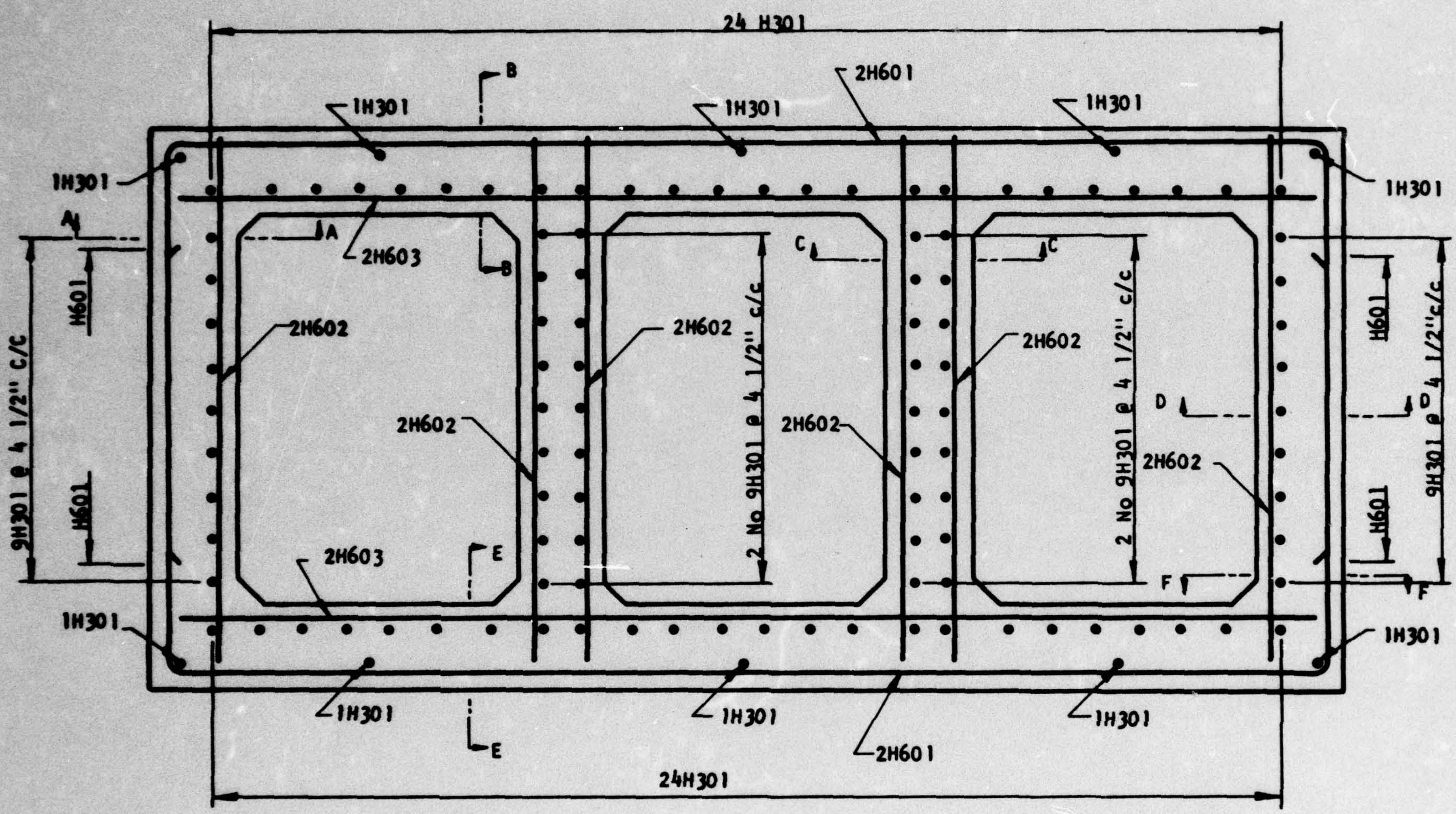
$$\text{Thrust} = C_s + C_c - T$$

$$\Sigma M_{centroid} = C_s \left(\frac{h}{2} - d'\right) + C_c \left(\frac{h}{2} - \frac{a}{2}\right) + T \left(d - \frac{h}{2}\right)$$

$$\beta_1 = 0.85 \text{ if } f'_c \leq 4{,}000 \text{ lb/in.}^2$$

$$\beta_1 = 0.85 - 0.05 \left(\frac{f'_c - 4{,}000}{1{,}000}\right) \text{ if } f'_c > 4{,}000 \text{ lb/in.}^2$$

FIG. 13 SUMMARY OF METHOD OF CALCULATION OF MOMENT-THRUST INTERACTION DIAGRAMS



Note: Clear Cover: $1\frac{1}{2}$" To All Side Faces

FIG. 14 ARRANGEMENT OF REINFORCEMENT IN SPECIMENS R2 AND R3

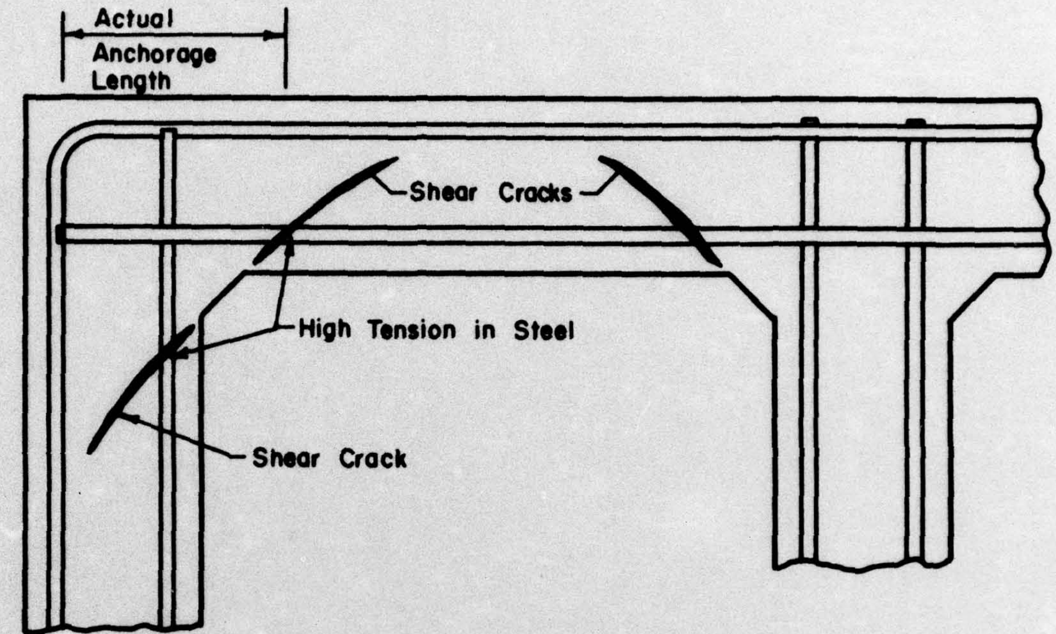


FIG. 15 LOCATIONS OF POTENTIAL SHEAR CRACKS RELATIVE TO ENDS OF MEMBERS

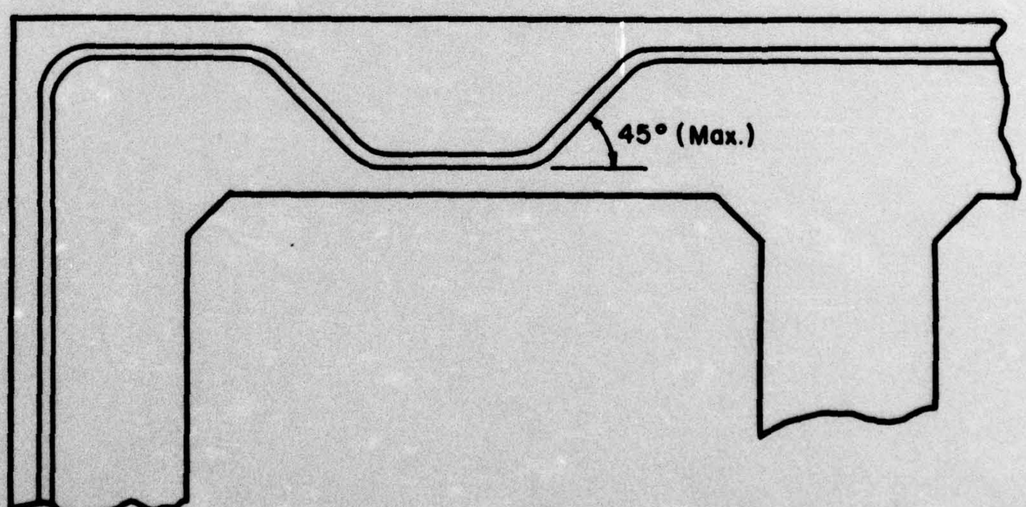


FIG. 16 INEFFECTIVE USE OF BENT-BAR REINFORCEMENT IN DEEP MEMBER